DESIGN EXAMPLE F-4

TEN-STORY CONCRETE FRAME AND SHEAR WALL BUILDING

Purpose. This example is presented to illustrate a procedure to evaluate an existing reinforced concrete structure, determine if it satisfies the acceptance criteria, and develop an upgrading concept for resistance to seismic forces.

Description of Structure. A 10-story office building (plus basement) with lateral force resisting systems consisting of reinforced concrete moment-resisting frames in the longitudinal direction and reinforced concrete shear walls in the transverse direction. The building was designed and built in the late 1960*s in accordance with the provisions of the 1964 Uniform Building Code (UBC). The earthquake design provisions are essentially identical to “Seismic Design for Building” (BDM) dated 15 March 1966 (TM 5-809-10/NAVDOCKS P-355/AFM 88-3, Chapter 13). These design provisions had not yet provided for concrete ductile moment— resisting space frames. However, the designer had provided some of the ductility requirements later adopted by the UBC and included in the April 1973 edition of the BDM. The ductility was provided using the concepts developed by Blume, Newmark, and Corning in “Design of Multistory Reinforced Concrete Buildings for Earthquake Motions,” Portland Cement Association, Skokie, Illinois, 1961. The structural design concepts are illustrated on sheets 2 through 5.

Construction Outline.

Roof:
- Built-up roofing.
- Reinforced lightweight concrete slabs, joists, and girders.
- Suspended ceiling.

Typical Floors:
- Reinforced lightweight concrete slabs, joists, and girders.
- Asphalt tile.
- Suspended ceiling.

Basement Floor:
- Reinforced concrete slab-on-grade.
- Asphalt tile.
- Suspended ceiling.

Foundation:
- Reinforced concrete mat.

Columns:
- Reinforced lightweight concrete.

Exterior Walls:
Reinforced concrete.

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Figure F-4. Building with concrete moment-resisting frames and shear wails. (Sheet 1 of 32)

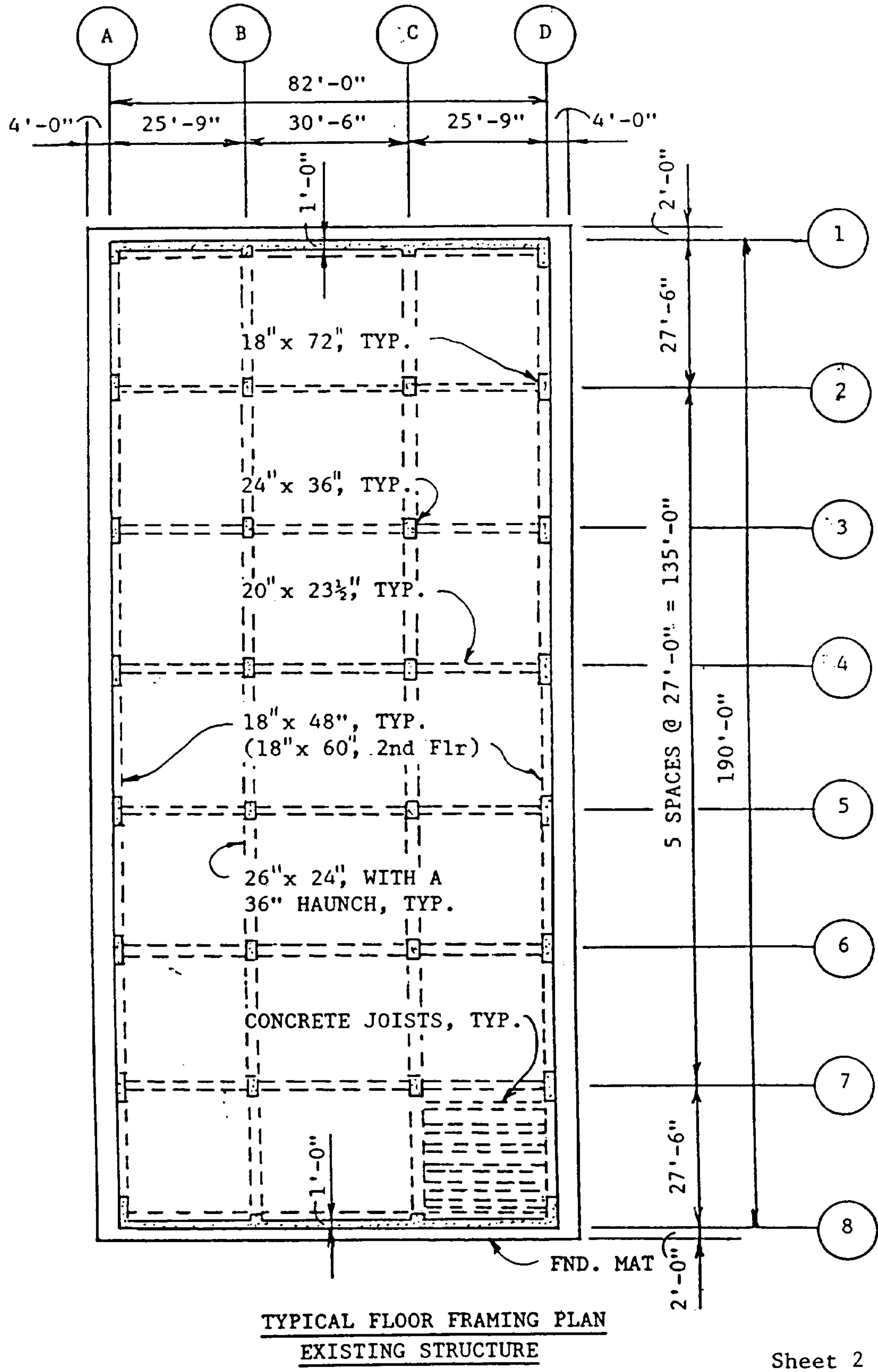
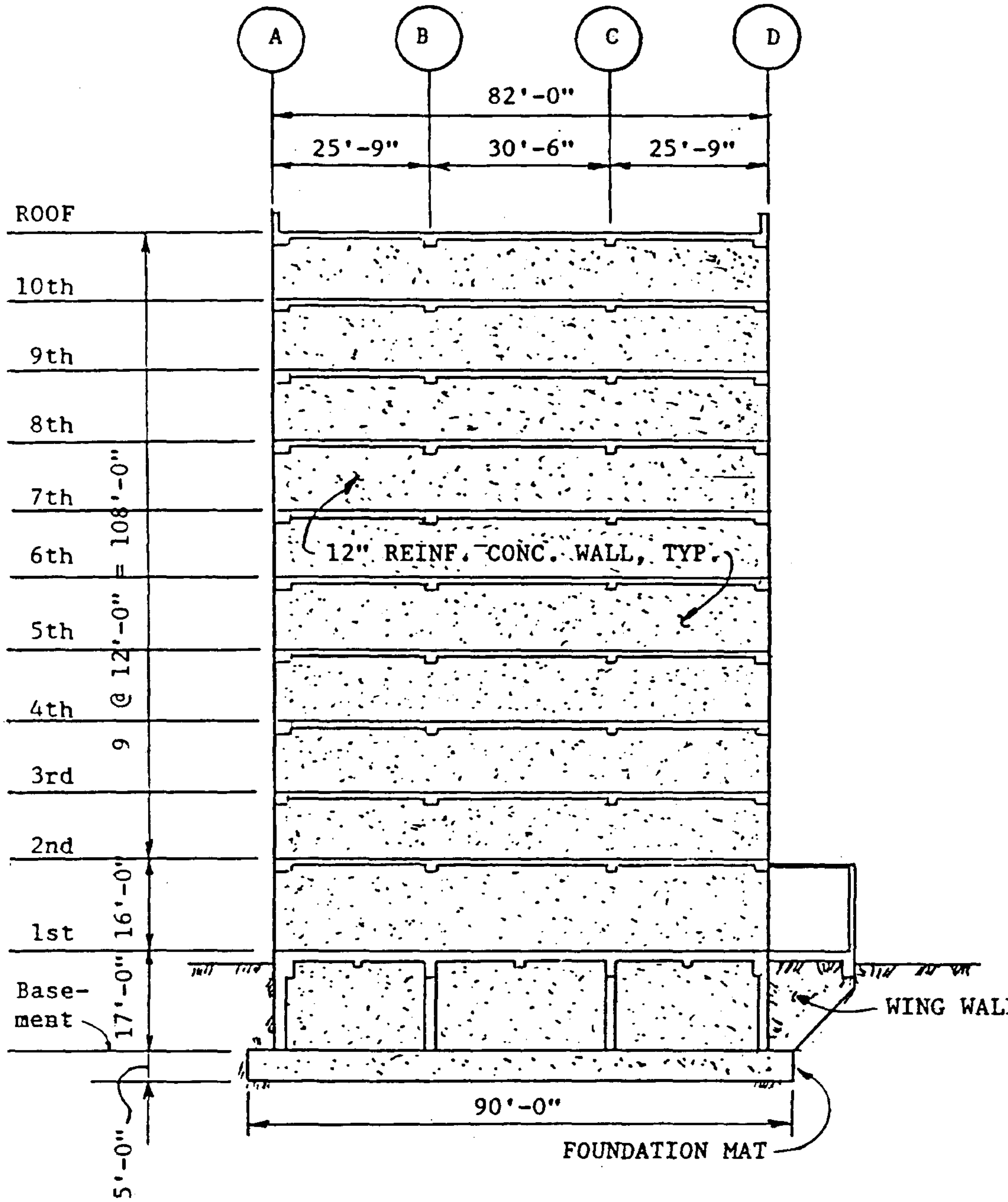


Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 2 of 32)



LINES 1 & 8 (EXISTING)

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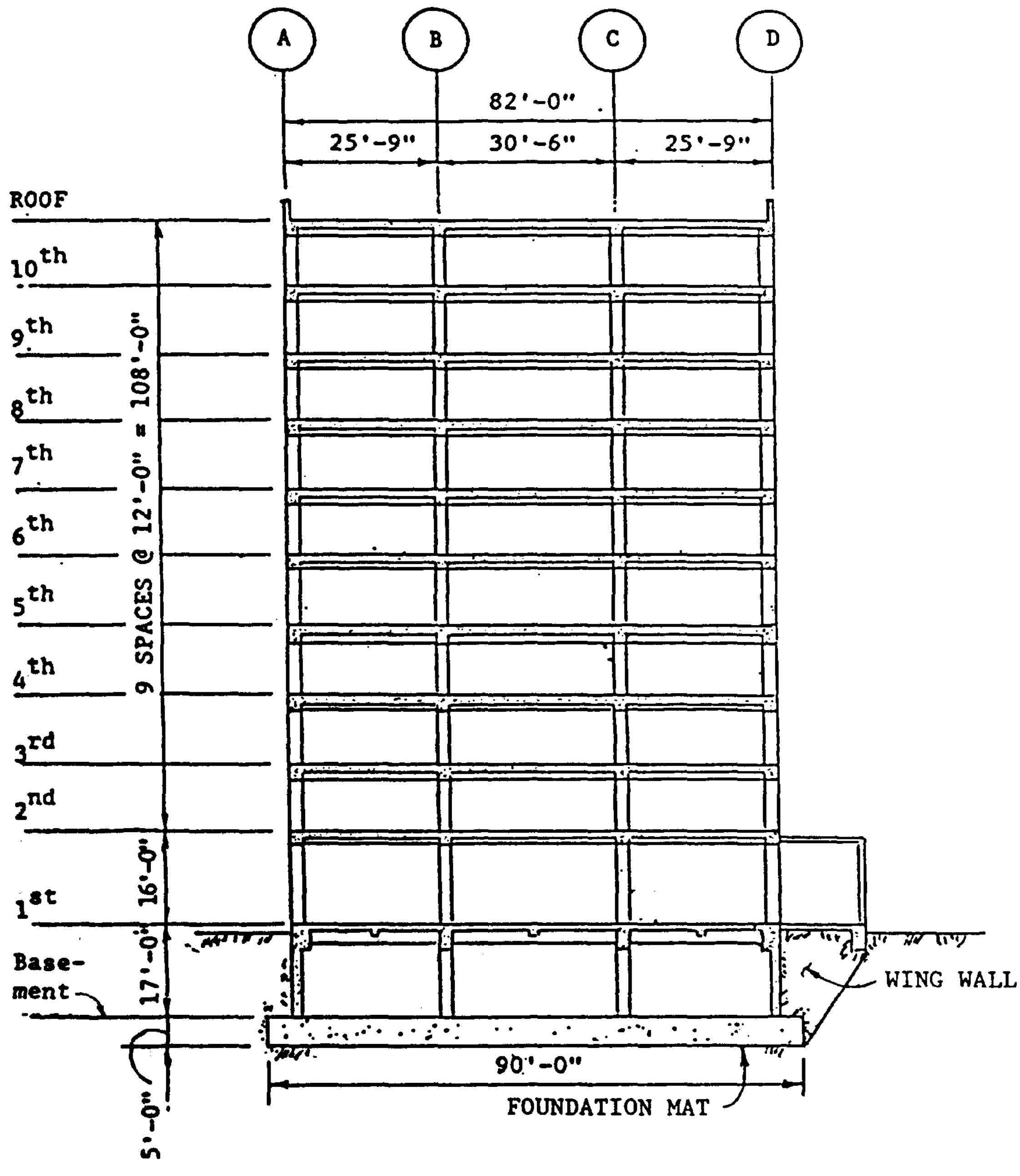
Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 3 of 32)



FRAME LINES 2, 3, 4, 5, 6 & 7
(EXISTING)

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 4 of 32)

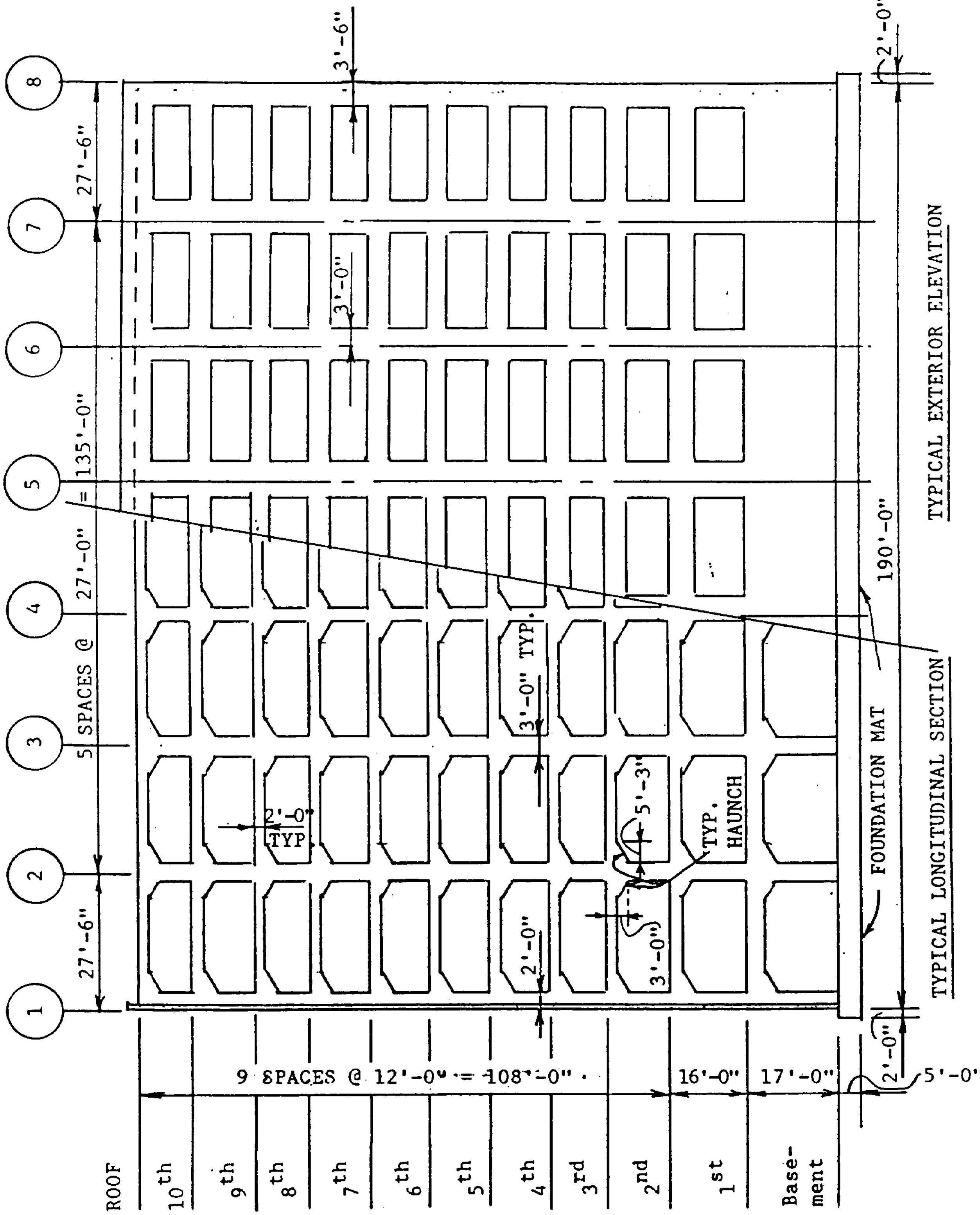

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 5 of 32)

Original Design. The original design for earthquake forces was based on the 1964 UBC (similar to 1966 BDM). The base shear was determined as follows:

$$V = ZKCW$$

where Z = 1.0 (seismic zone coefficient)
K = 1.0 (building systems coefficient
$C = 0.05/T^{1/3}$

In the traverse direction, $T = 0.05\ h/d^{1/2} = 0.68$ sec
$C = 0.057$
In the longitudinal direction, $T = 0.1\ N = 1.0$ sec
$C = 0.05$

The weight $W = 32{,}600^k$ on the basis of regular weight concrete. Reinforced concrete design criteria were based on working stress design (WSD).

Design base shear:

Transverse = $1 \times 1 \times 0.057 \times 32{,}600 = 1860^k$
Longitudinal = $1 \times 1 \times 0.05 \times 32{,}600 = 1630^k$

Note: Due to "fast-tracking" of this building, the foundations were designed and under construction prior to completion of superstructure design. Because the above building weight would have overloaded the foundation soils, it was decided to use lightweight concrete *for* the frames and floors but not for the shear walls. This reduced the weight to $27{,}040^k$ and increased the effective base shear coefficients to:

$V/W = 0.069$ transverse
$V/W = 0.060$ longitudinal

In addition to the minimum requirements of the code, the engineer decided to supply additional detailing to provide ductility in accordance with the concepts developed by Blume, Newmark, and Corning. This included additional column ties (or hoops) in the column and in the beam-column joint zone to provide for confinement.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 6 of 32)

Site Response Spectra. Site response spectra, which are used for the preliminary evaluation, the detailed analysis, and the upgrade concept, were developed in accordance with the procedure in chapter 3 of the SDG:

Building Classification: Others

Ground Motion Spectra: ATC 3-06 Map Contour Level
$A_a = A_v = 0.30$

Soil Classification: $S_i = 1.0$ (Type S1)

Earthquake I
Damping = 5%, D.F. = 1.00 (SDG table 3-7)
$A_a = A_v = 0.14$ kg (Design Ground Motion, SDG table 3-4)
S_a = D.F. $(1.22A_vS_i)/T = 0.342g/T$ less than D.F. $(2.5)A_a = 0.70$ g max

EQ-II/EQ-I = 0.70/0/35 = 2.0

The resulting spectra are shown in sheet 8.

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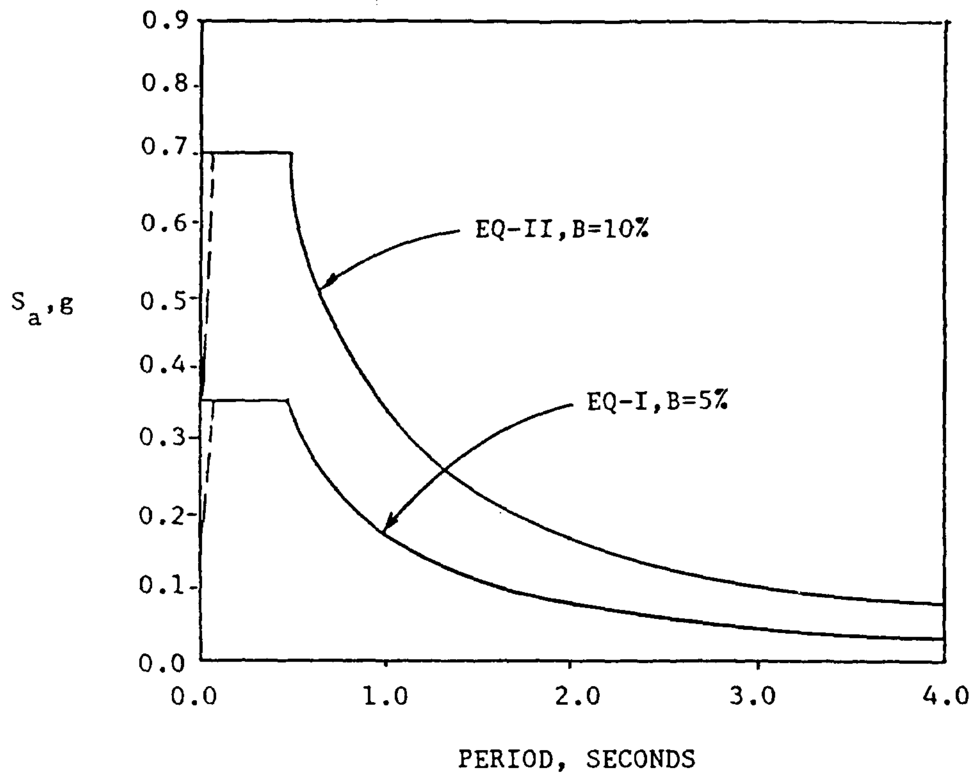
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 7 of 32)

DESIGN RESPONSE SPECTRA FOR EQ-I AND EQ-II



EQ	B		PERIOD							
			0.0	.488	.80	1.0	1.5	2.0	3.0	4.0
I	5%	S_a, g	.14	.35	.214	.171	.114	.085	.057	.043
II	10%	S_a, g	.35	.70	.427	.342	.228	.171	.114	.086
		S_d, in	0	1.63	2.68	3.35	5.02	6.70	10.04	13.47

* SPECTRAL DISPLACEMENT $S_d = S_a(T/2\pi)^2 g$

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 8 of 32)

PRELIMINARY Evaluation. A rapid evaluation of the structure was made using available data. For the longitudinal direction, the capacity was approximated by using the design base shear and assuming yield was at two times design. For the transverse direction, the capacity was approximated from the strength and area of the shear walls. Calculations are shown on sheets 10 and 11. The capacity spectrum method (sheet 12) was used to approximate damage. Over 100 percent for transverse, 70 percent for longitudinal, and 99 percent for combined (total) damage due to EQ-II. The results of the preliminary evaluation indicate that the structure will be substantially damaged by EQ-II; however, for a smaller earthquake (e.g. , EQ-I) the results of the evaluation indicate that the structure would remain essentially elastic. Because of the size and value of the building, it was decided that a detailed analysis would be warranted to more accurately determine how the structure would perform under EQ-II loading.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 9 of 32)

RAPID EVALUATION PROCEDURE:

Longitudinal Direction: Moment frame

Effective design base shear coefficient at WSD

$V/W = 0.067$

Yield Capacity

Assume yield at 2 x design

$C_B = 2 \times 0.067 = 0.134$

Estimate period: $T = 0.1N = \underline{1.0}$ sec

$$2\sqrt{f'_c} = 2\sqrt{3000} = 110 \text{ psi}^*$$

$$pf_y = \frac{2\text{x}0.20}{12\text{x}12} \times 40{,}000 = \underline{111} \text{ psi}$$

Total 221 psi

$V_{CAPACITY} = 0.221 \times 23{,}616 = 5220^k$

*May be 5000 psi concrete, see detailed analysis

$\Delta_R = P.F._{ROOF} \times S_d = 1.5S_d = 2.1$ inches

Average interstory drift ratio

$\Delta_R/H = 2.1/124 \times 12 = 0.0014$

Ultimate Capacity

Assume: ULTIMATE CAPACITY = 1.5 x YIELD
ULTIMATE DISPLACEMENT = 4 x YIELD

$S_{aU} = 1.5 \times 0.168g = \underline{0.252}g$

S_{dU} 4 x 2.1 inches = $\underline{8.4}$ inches

$T_U = 2\pi\sqrt{S_d/S_a g} = \underline{1.84}$ sec

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 10 of 32)

RAPID EVALUATION PROCEDURE (continued)

Transverse Direction: Shear Walls

Effective design base shear at WSD: $V = 1860^k$; $V/W = 0.076$
Area Shear Walls: (82' x 1')2 x 144 = 23,616 square inches
Wall Capacity: 12" wall w/#4 at 12"ef,ew

$$2\sqrt{f'_c} = 2\sqrt{3000} = 110 \text{ psi}^*$$
$$pf_y = \frac{2x0.20}{12x12} \times 40{,}000 = \underline{111} \text{ psi}$$
$$\text{Total} \qquad 221 \text{ psi}$$

$$V_{CAPACITY} = 0.221 \times 23{,}616 = 5220^k$$

*May be 5000 psi concrete, see detailed analysis

Yield Capacity

$$C_B = V_{CAP}/W = 5220/24500 = 0.213$$

Est. Roof Displ.: Shear $\Delta = \frac{VH}{AG} \simeq 150 \text{ psi (avg)} \times \frac{124x12}{1.2x10^6} = \underline{0.2"}$

Bending: $\Delta = \frac{PL^3}{3EI} \simeq \frac{2/3(5220)x124^3x12}{3(3x10^3x144)x46000x2} = \underline{0.66"}$

$\Delta_R \simeq \underline{0.9"}\pm$

If T = 0.68 sec (sheet 6) and $S_a = \frac{C_B}{0.8} = \underline{0.27}g$:
$S_d = \left(\frac{T}{2\pi}\right)^2 S_a \times g = 1.22"$; $\Delta_R \simeq 1.3S_d = \underline{1.5}"$

Assume Δ_R = 1.0" (includes rocking added to 0.9")
$S_d = 1.0 \div 1.3 = \underline{0.77}"$ and $S_a = \underline{0.27}g$
$T = 2\pi\sqrt{S_d/S_a x g} = \underline{0.62}$ sec

Ultimate Capacity

Assume $S_{aU} = 1.25 \times 0.27 = \underline{0.34}g$
$S_{dU} = 4 \times 0.77 = \underline{3}"$
$T = \underline{0.95}$ sec

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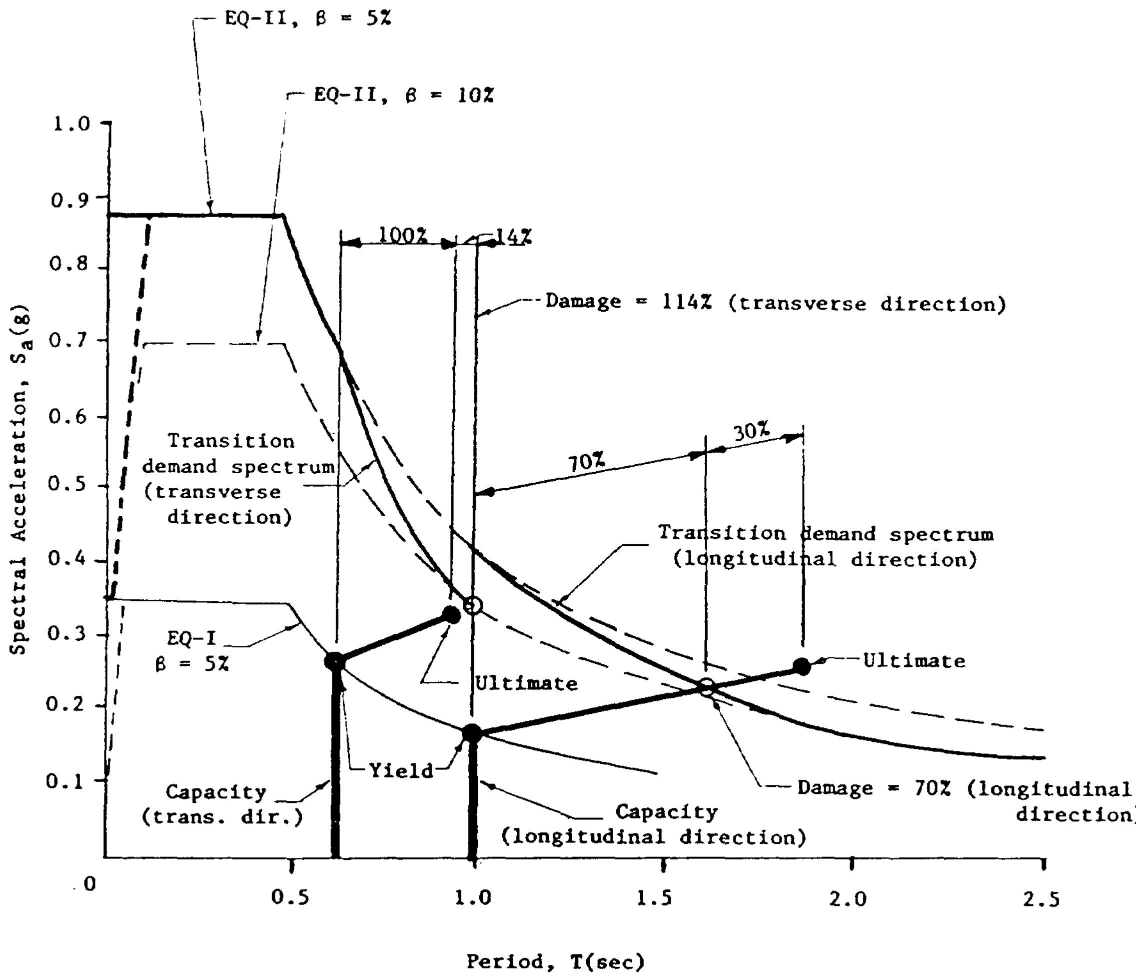
Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 11 of 32)



<u>Rapid Evaluation</u>

Transverse direction:	114% damage
Longitudindal direction:	70% damage
Total damage:	2/3(114) + 1/3(70) = 99%

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 12 of 32)

Acceptance Criteria (para 5-2). The acceptance criteria for existing buildings are those presented for the post yield analysis for EQ-II in the SDG with latitude allowed under certain conditions.

Conforming/Nonconforming Systems and Materials. The reinforced concrete moment frames do not strictly conform to current standards; however, some ductility provisions were incorporated into the design and the current condition of the building is good. Therefore, the structure will be considered essentially conforming with some latitude allowed in the acceptance criteria.

Method 1. Elastic Analysis Procedure (Refer to SDG paras 4-4c and 5-5a).

Classification: Other Buildings

Loading Combination: DL + 0.25LL + 1.0 EQ

Ultimate Strength Capacities: ACI 318 Strength Design

Inelastic Demand Ratios: (table 5-1)

	Nonduct.	Ductile	Avg.
Reinf. Conc. Frames			
Columns	1.25	1.75	1.5
Beams	1.75	3.00	2.4
Reinf. Conc. Shear Walls			
Single Curtain Reinf.	Shear-1.50, Flexure-2.0		
Double Curtain Reinf.	Shear-1.75, Flexure-3.0		
Reinf. Conc. Diaphragms	Shear-1.75, Flexure-2.0		

Material Properties

Lightweight Concrete	f'_c = 3750 psi
Regular Weight Concrete	f'_c = 4000 psi
Reinforcement	F_y = 40 ksi

Story Drift Limitation: 0.015 x Story Height

Method 2. Capacity Spectrum Method (Refer to SDG paras 4-4d and 5-5b). If the acceptance criteria of Method 1 are not satisfied, the structure will be analyzed in accordance with Method 2 prior to developing a seismic upgrading concept.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 13 of 32)

Detailed Structural Analysis.

Method 1. The existing structure was analyzed with the aid of a computer. Gross concrete section properties of the girders and columns were used for the moment frame. For simplicity the haunches were neglected. Also, the stiffening effects of the floor system were ignored in the mathematical model. It is assumed that the contribution of these items to stiffness are relatively small and are balanced out by neglecting the reduced stiffness effects of nominal "cracked" section properties.

The mathematical model was subjected to an elastic modal analysis using the design response spectrum for EQ-II, 10 percent damped, shown on sheet 8. The results of the analysis gave the following:

	Transverse	Longitudinal
Fundamental Period (sec)	0.46	0.80
Base Shear, 1st Mode (kips)	13,980	9,520
Base Shear, RSS (3 modes)	14,485	9,764
Roof Displacement (ft)	0.172	0.292
Roof Acceleration, 1st mode	1.00g	0.556g
Roof Acceleration, RSS (3 modes)	l.l0g	0.656g

The results indicate that the structure is relatively stiff, such that the calculated periods are shorter than the empirical periods used in the original design (sheet 6). The EQ-II shear forces are 7.5 times design in the transverse direction and 5.8 times in the longitudinal direction.

Sample IDR* 's of the most critical elements follow:

	Calculated	Allowable
Transverse Shear Walls	IDR = 2.94	1.75 N.G.
Longitudinal frame, girder bending	IDR = 2.3	2.4 O.K.
Longitudinal frame, column bending	IDR = 2.0	1.5 N.G.

*IDR*s are calculated by dividing the computer calculated force by the strength capacity for each element.

The conclusions of the Method I detailed evaluation indicate that the existing building does not conform to the acceptance criteria. However, the results are based on a gross concrete section model. With large overstresses it is likely that the period will lengthen (due to cracked concrete) and reduce the effective earthquake forces on the building. It should also be noted that as some elements yield, additional load will be distributed to other members. In the elastic model, the transverse interior frames only take about 3 percent of the lateral forces. However, if the shear walls yield, the frames can

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 14 of 32)

contribute some backup resistance. In order to get a better feel for the inelastic response of the building a Method 2 analysis was done.

Method 2. The Capacity Spectrum Method uses a step-by-step, pseudo-inelastic approach to approximate the inelastic capacity of the structure. This capacity is compared by means of a graphical procedure to the demands of the EQ-II response spectrum. Guidelines for this procedure are presented in the SDG, para 5-5.

For this example, the pseudo-inelastic analysis consisted of consecutive elastic analyses of an initial mathematical model of the structure that was modified in an iterative fashion to include the results of the previous analyses and loaded incrementally. The process began by defining the initial 2-D model as is typically done for any computerized elastic analysis (e.g., the analysis used in Method 1, sheet 14). In addition, beam yield strengths for positive and negative bending, beam shear capacities, and beam and column gravity induced forces were computed. For beams to be subjected to negative seismic bending, a seismic reserve capacity equal to beam negative yield strength less gravity moment at the face of support was computed. For beams to be subjected to positive seismic bending, the seismic reserve capacity equals the beam positive yield strength plus the gravity moment at the face of support. For columns, P-M interaction diagrams were used to aid in identifying load capacities as shown on sheet 17.

The incremental loading regimen commenced with the application of the EQ-II Spectrum (sheet 8) loading to the 2-D mathematical model of the initial structure. Seismic member forces derived from this analysis were compared to member seismic reserve capacities to identify the first set of plastic hinges to form and to obtain the maximum member overstress factor. The initial loading, $S_{aII,}$ divided by this overstress factor defines the load, Say, at first yielding as well as the seismic member forces associated with first yielding.

For the second step, the mathematical model was altered to include pinned member ends which reflected the first set of plastic hinge locations. This model was subjected to a small, monotonic, incremental load, S_{aY}, and reanalyzed using the same elastic computer program. The new set of seismic member forces obtained from this was added to those corresponding to first yielding and this sum was again compared to the member seismic reserve capacities; thus a second set of plastic hinges could be identified. Subsequent analyses were performed identically, each time including the new set of plastic hinges in the previous model and comparing the summation of the member forces of previous analyses to the initial member seismic reserve capacities. The method of superposition of the incremental loads are illustrated in sheets 19 and 20 of Figure E-3 of the SDG.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 15 of 32)

Longitudinal Direction. The results for the longitudinal direction are shown on sheets 18, 19, and 20. Sheet 18 shows the sequence of plastic hinges. Sheet 19 shows the relationship between V, $)_R$, C_B, S_a, S_d and T, and plots the capacity curves. Sheet 20 shows the graphical solution for the capacity spectrum method.

From sheet 20 it appears that the structure, in the long direction, can survive EQ-II without collapse and that it will remain essentially elastic for EQ-I. The capacity curve crosses the demand curve (EQ-II) at approximately $S_a = 0.244g$ and T 1.44 sec.

$$S_d = (T/2\pi)^2 S_a g = 4.95''$$

$$\Delta_R = 1.30 S_d = 6.42''$$

$\Delta_R/H = 6/124 \times 12 = 0.0043$, Avg. drift ratio

If worst story = 2 x Avg

Max. story drift ratio = $0.008 < 0.015$ O.K.

Although these analytical results are encouraging, the "survival" of the building against collapse for EQ-II should be considered marginal. More conservatism in modeling, application of the modal story force, or consideration of possible beam/column deterioration due to repetitive cycling of the inelastic rotation would tend to depress the capacity curve of sheet 20 below the demand spectrum of EQ-II.

Transverse Direction. The detailed evaluation for the transverse direction was not in the scope of this example. Because the calculated period of the structure is shorter than the one obtained by the empirical formula, it appears that the performance of the structure will be worse than approximated in the rapid evaluation. However, it should be noted that a detailed evaluation of the shear wall energy absorbing capabilities after initial yielding may show that the performance characteristics of the transverse direction are better than anticipated.

Results of Detailed Structural Analysis. Although the results indicate that the building may be severely damaged if subjected to the EQ-II earthquake, the overall performance characteristics are relatively good considering the age (pre-1973) and type of construction (reinforced concrete frame). It appears that the building will perform in an essentially elastic manner for EQ-I but compliance with the acceptance criteria for EQ-II may be marginal. It is therefore recommended that upgrade concepts be developed and that a cost-benefit study be made to determine priorities for upgrading.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 16 of 32)

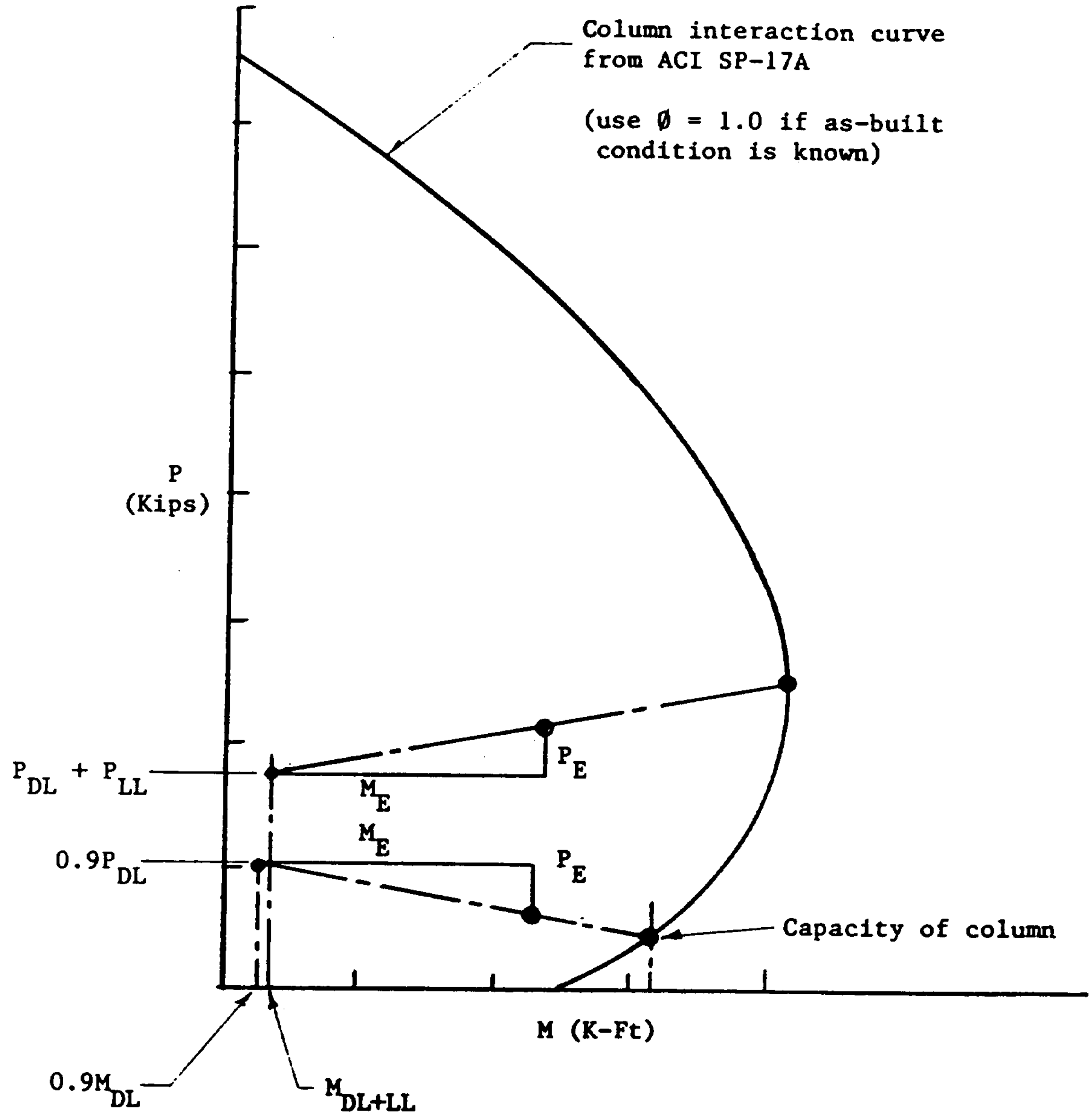

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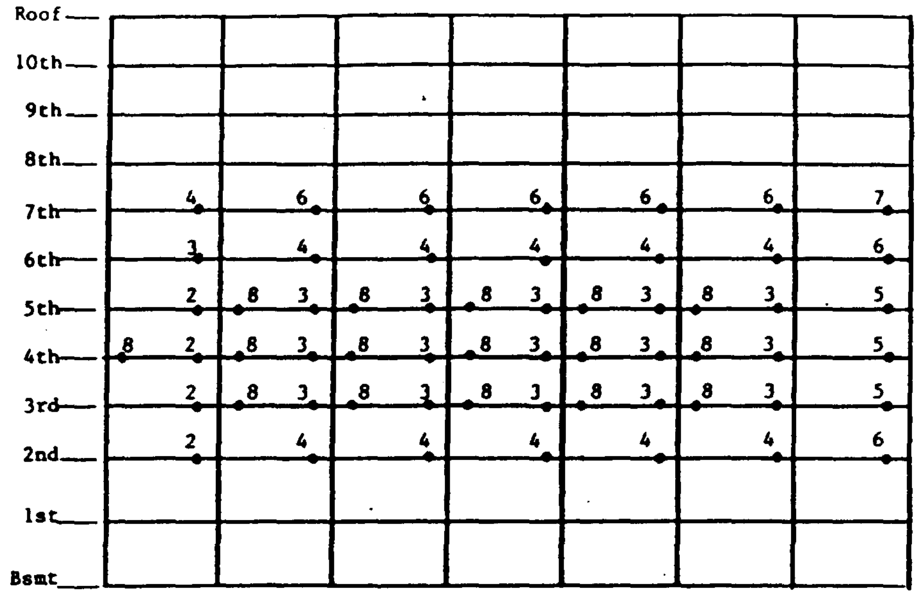
Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 17 of 32)

FRAMES B & C



Load cycle	1	2	3	4	5	6	7	8	9
S_a*	.144g	.03g	.01g	.01g	.01g	.01g	.01g	.01g	.01g

* Load increment

FRAMES A & D

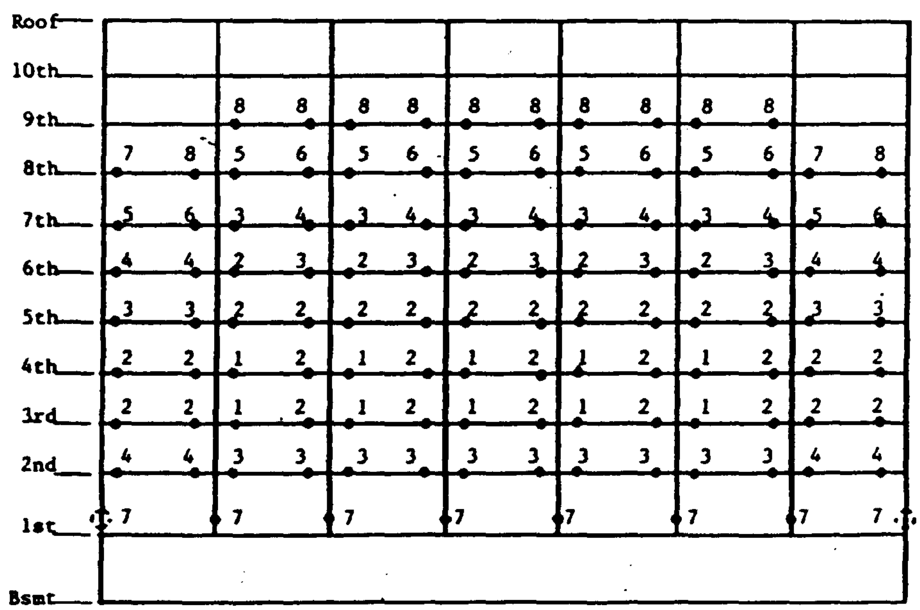


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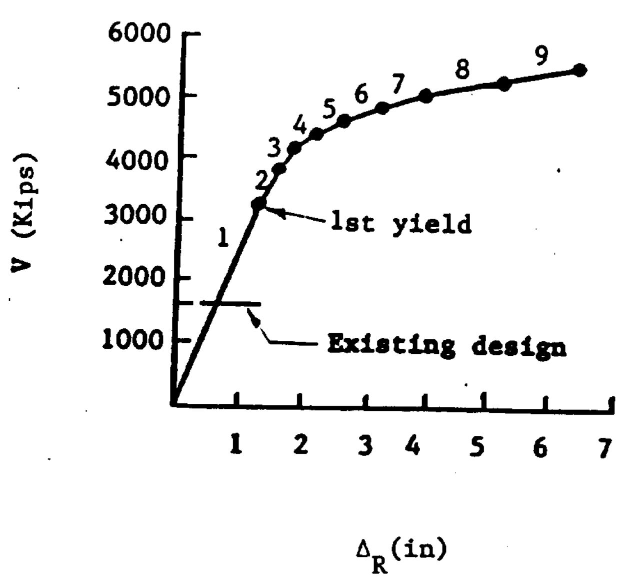
Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 18 of 32)

EQ-II: CAPACITY SPECTRUM ($W = 27{,}000^K$)

INCR.	S_{ai}	ΣS_a	S_{di}	ΣS_d	Δ_{Ri}	$\Sigma \Delta_R$	V	C_B	T	$\frac{\Sigma \Delta_R}{\Sigma S_d}$	$\frac{C_B}{\Sigma S_a}$
1	0.144g	0.144g	0.94"	0.94"	1.25	1.25	3256	0.120	0.817	1.33	0.83
2	0.03	0.174	0.24	1.18	0.31	1.56	3931	0.147	0.832	1.32	0.84
3	0.01	0.184	0.14	1.32	0.17	1.73	4154	0.153	0.856	1.31	0.83
4	0.01	0.194	0.27	1.59	0.32	2.05	4379	0.162	0.915	1.29	0.84
5	0.01	0.204	0.41	2.00	0.50	2.55	4606	0.170	1.001	1.28	0.83
6	0.01	0.214	0.48	2.48	0.60	3.15	4830	0.179	1.088	1.27	0.84
7	0.01	0.224	0.54	3.02	0.69	3.84	5051	0.187	1.174	1.27	0.83
8	0.01	0.234	0.92	3.94	1.23	5.07	5285	0.196	1.312	1.29	0.84
9	0.01	0.244	1.01	4.95	1.85	6.42	5517	0.204	1.440	1.30	0.84



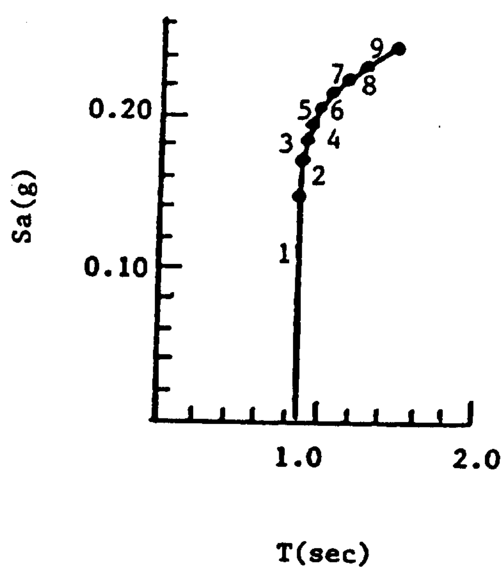


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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 19 of 32)

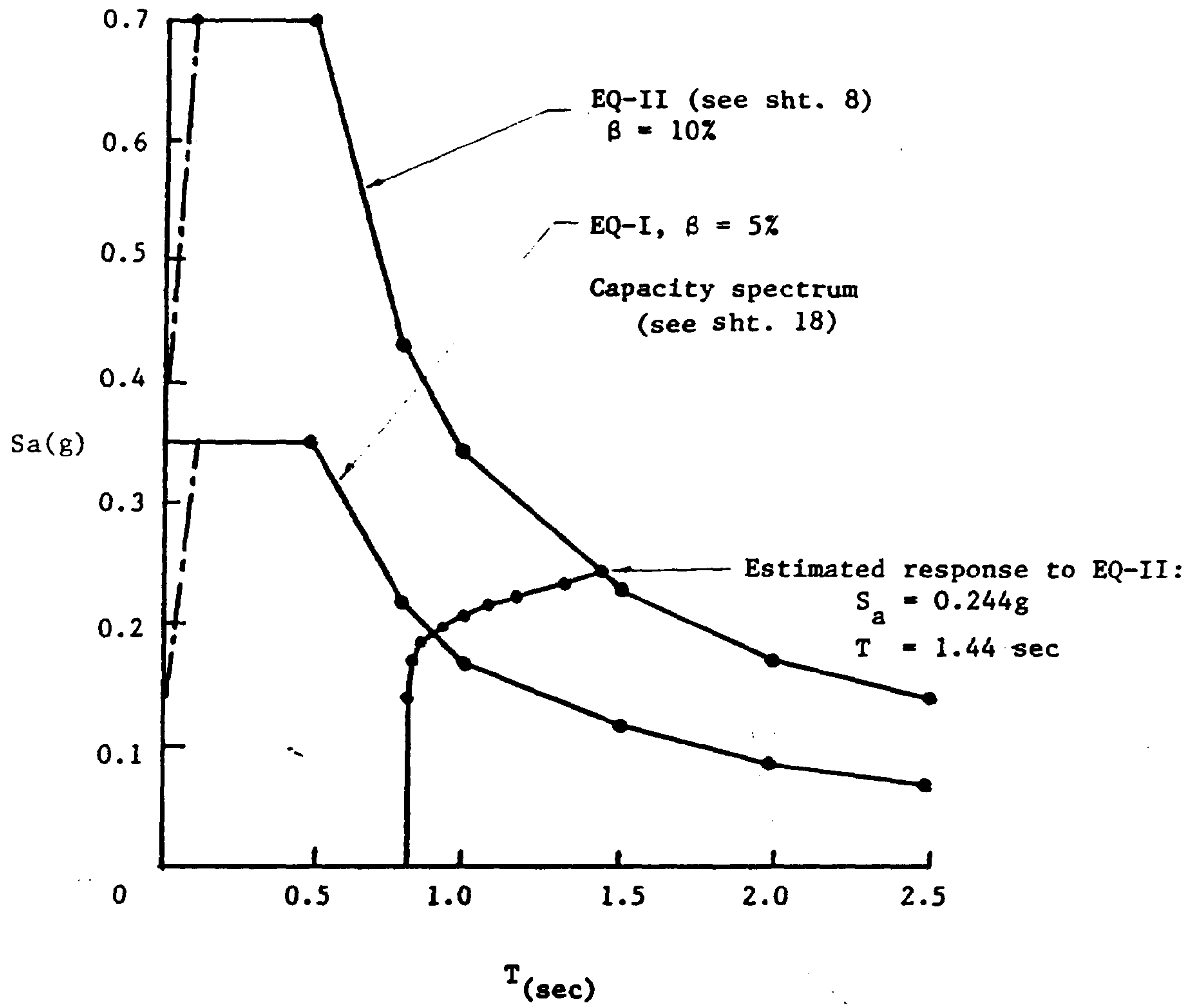

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 20 of 32)

Development of Seismic Upgrade.

Structural Upgrading Concept. The recommended upgrading concepts include the addition of interior cast-in-place reinforced concrete walls, to resist the transverse seismic forces and reduce the diaphragm stresses, and the placement of cast-in-place reinforced concrete panels in alternate window openings in the exterior concrete frames to resist the seismic forces in the longitudinal direction. For plans and elevations of the upgrade concept see sheets 22, 23, and 24.

Confirmation Analyses. A modal analysis of the modified structure was made with the aid of a general computer program for the static and dynamic analyses of frame and shear wall three-dimensional buildings for both the transverse and longitudinal directions. The program assumes rigid diaphragms and the roof and the floor diaphragms of this modified structure essentially meet the requirements of this assumption. The mathematical model was assumed fixed at the first floor level. The dynamic modal responses are indicated on sheets 25 and 26.

Structural Member Responses. Sheets 27 and 28 indicate the SRSS of modal responses for representative structural members in the transverse and longitudinal directions. The accidental torsion responses were calculated as described for design example F-2 and are given on sheet 29. A check of selected structural elements for compliance with the acceptance criteria is given on sheets 30 and 31.

Torsional Forces. Due to the symmetry of the structure lateral load resisting system there is no “calculated torsion.” The “accidental" torsion is the story shear times the nominal eccentricity of 5 percent of the maximum building dimension. The torsional forces for the roof and the floors are distributed to the lateral force resisting elements in accordance with the method illustrated in the BDM Example A-3 and added to the forces from the dynamic analysis.

Overturning Forces. A check of the overturning forces due to EQ-II resulted in no instability of the structure as a whole. The soil pressure at the toe of the foundation mat due to DL + 0.25 LL + EQ-II forces in the transverse direction exceeds more than twice of the allowable design soil pressure when based on a. triangular distribution of the soil pressure. Soil pressure under a rectangular distribution assumption results in a soil pressure less than twice the allowable design pressure. A Soil Engineering firm should be consulted to reevaluate the allowable soil pressure and the shape of the soil distribution pressure under dynamic loadings.

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 21 of 32)

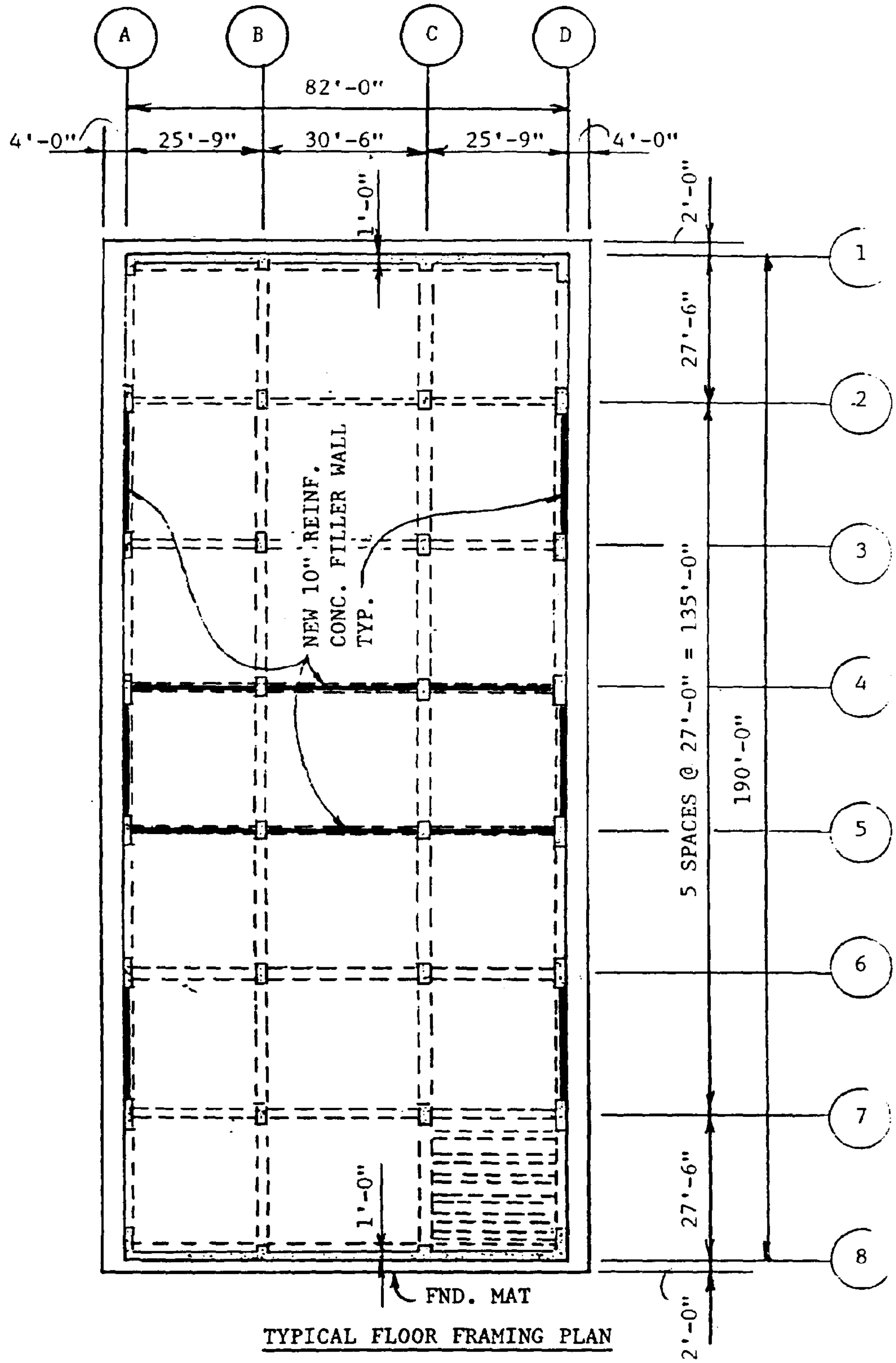

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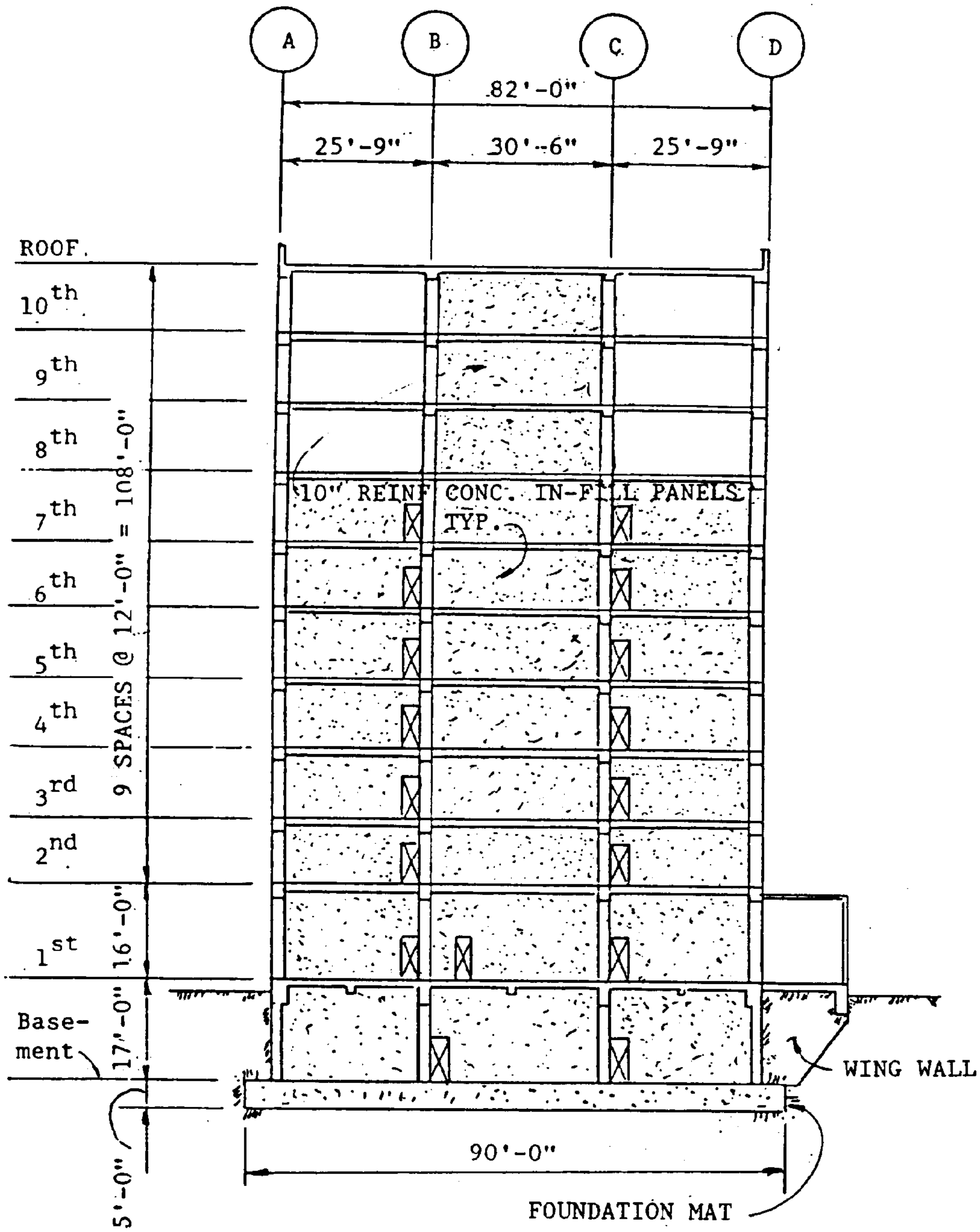
Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 22 of 32)



SHEAR WALL LINES 4 & 5

(NEW)

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 23 of 32)

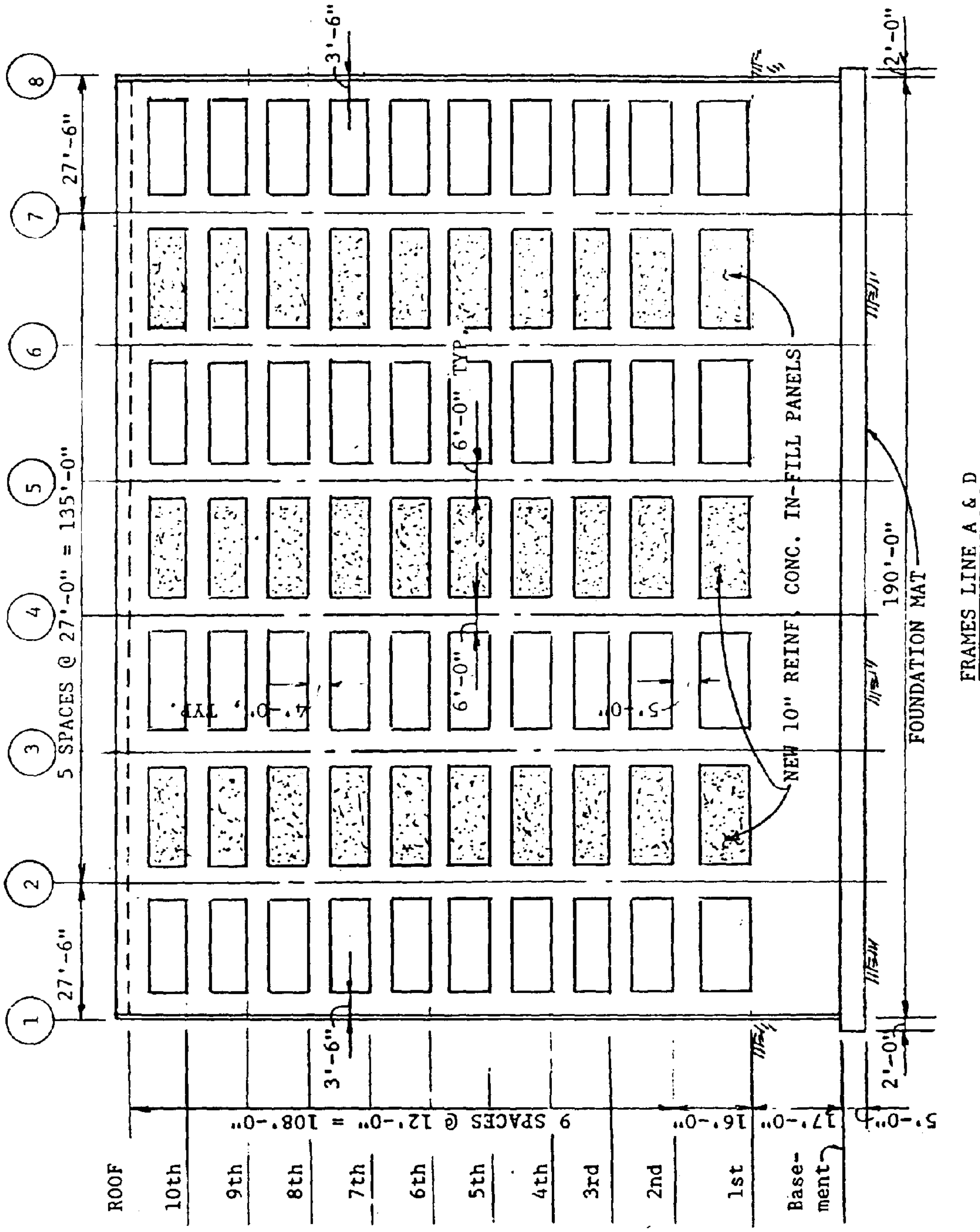

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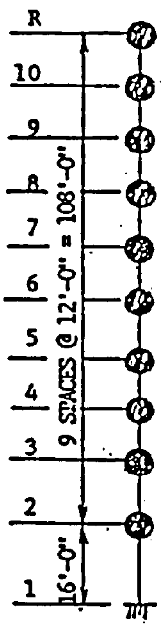
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 24 of 32)

EQ II STRUCTURAL RESPONSE - SRSS



MASS (Kips-Sec/Ft.)	STORY LOAD F_x ** (kips)	STORY SHEAR V_x** (kips)	DISPLACEMENT Δ_x ** (Feet)	STORY DRIFT $\delta\Delta_x$ ** (Feet)
90.02	3033		0.202	
		3033		0.014 0.120 *
86.87	2576		0.188	
		5594		0.017
86.87	2284		0.171	
		7783		0.020
89.06	2136		0.152	
		9704		0.022
91.25	1995		0.130	
		11,394		0.024
91.25	1786		0.106	
		12,509		0.025
91.25	1577		0.082	
		13,945		0.024
91.25	1399		0.057	
		14,810		0.022
91.25	1122		0.035	
		15,406		0.019
101.48	751		0.016	
910.55		15,760***		0.016 0.160*

MATHEMATICAL MODEL

T_1 = 0.511 Sec.

T_2 = 0.144 Sec.

T_3 = 0.073 Sec.

* MAXIMUM ALLOWABLE STORY DRIFT = 0.010H

** $F_x = \left[\Sigma F_{xm}^2\right]^{\frac{1}{2}}$ $\quad V_x = \left[\Sigma V_{xm}^2\right]^{\frac{1}{2}}$

$\Delta_x = \left[\Sigma \Delta_{xm}^2\right]^{\frac{1}{2}}$ $\quad \delta\Delta_x = \left[\Sigma \delta\Delta_{xm}^2\right]^{\frac{1}{2}}$

*** $C_b = V_b \div \Sigma W = 0.537$

LONGITUDINAL DIRECTION

W = 910.55 x 32.2 = 29,320 kips

NOTE: $\Sigma F_x \neq V_x$ DUE TO HIGHER MODE PARTICIPATION EFFECTS ON FORCES.

$\Sigma\delta\Delta_x \simeq \Delta_x$ BECAUSE HIGHER MODE PARTICIPATION EFFECTS ON DISPLACEMENT ARE NEGLIGIBLE FOR THIS BUILDING. (i.e. RSS DISPLACEMENTS ARE ESSENTIALLY EQUAL TO 1st MODE DISPLACEMENT.)

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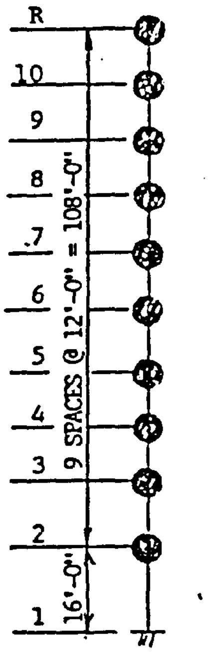
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 25 of 32)

EQ II STRUCTURAL RESPONSE - SRSS



MASS (Kip-Sec./Ft.)	STORY LOAD F_x ** (kips)	STORY SHEAR V_x** (kips)	DISPLACEMENT Δ_x** (Feet)	STORY DRIFT $\delta\Delta_x$** (Feet)
90.02	3286		0.105	
		3286		0.01 0.120*
86.70	2655		0.093	
		5917		0.012
86.70	2449		0.081	
		8039		0.013
89.06	2058		0.069	
		9815		0.011
91.25	1909		0.057	
		11,363		0.011
91.25	1731		0.046	
		12,662		0.011
91.25	1571		0.035	
		13,724		0.010
91.25	1413		0.025	
		14,565		0.009
91.25	1163		0.016	
		15,182		0.008
101.48	824		0.008	
910.55		15,586 ***		0.008 0.160*

* MAXIMUM ALLOWABLE STORY DRIFT = 0.010H

MATHEMATICAL MODEL

T_1 = 0.352 Sec.

T_2 = 0.103 Sec.

T_3 = 0.054 Sec.

** $F_x = \left[\sum (F_{xm})^2\right]^{\frac{1}{2}}$ $\qquad V_x = \left[\sum V_{xm}^2\right]^{\frac{1}{2}}$

$\Delta_x = \left[\sum (\Delta_{xm})^2\right]^{\frac{1}{2}}$ $\qquad \delta\Delta_x = \left[\sum \delta\Delta_{xm}^2\right]^{\frac{1}{2}}$

*** $C_b = V_b \div \Sigma W = 0.537$

TRANSVERSE DIRECTION

W = 910.55 x 32.2 = 29,320 kips

NOTE: $\Sigma F_x \neq V_x$ DUE TO HIGHER MODE PARTICIPATION EFFECTS ON FORCES.

$\Sigma \delta\Delta_x \approx \Delta_x$ BECAUSE HIGHER MODE PARTICIPATION EFFECTS ON DISPLACEMENT ARE NEGLIGIBLE FOR THIS BUILDING. (i.e. RSS DISPLACEMENT ARE ESSENTIALLY EQUAL TO 1st MODE DISPLACEMENT.)

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 26 of 32)

EQ II ELEMENT FORCES: TRANSVERSE DIRECTION - WALLS 4 & 5

SEISMIC RESULTS FROM COMPUTER ANALYSIS.
UNITS ARE KIPS AND KIP-FT.

	SHEAR	MOMENT
Roof		
	329	
10th		3943
	690	
9th		12224
	1130	
8th		25759
	2778	
7th		58983
	2969	
6th		94367
	3132	
5th		131525
	3260	
4th		169990
	3335	
3rd		209139
	3361	
2nd		248405
	3269	
1st		299234

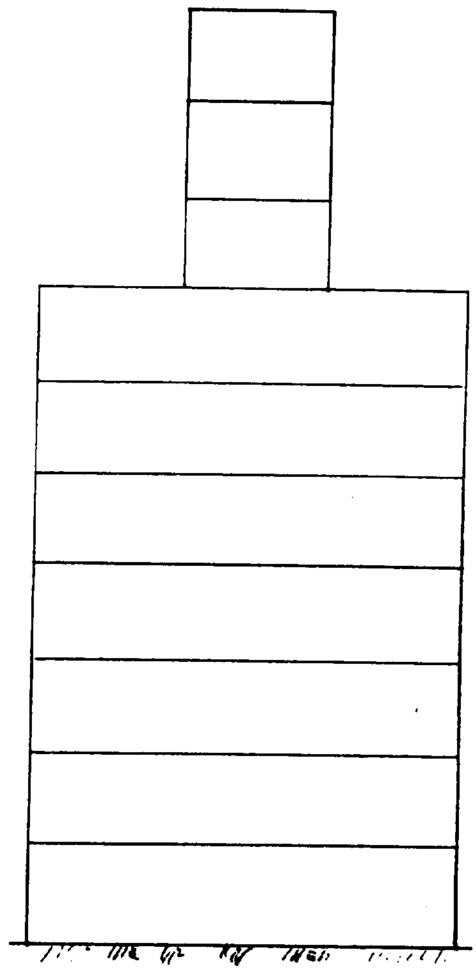

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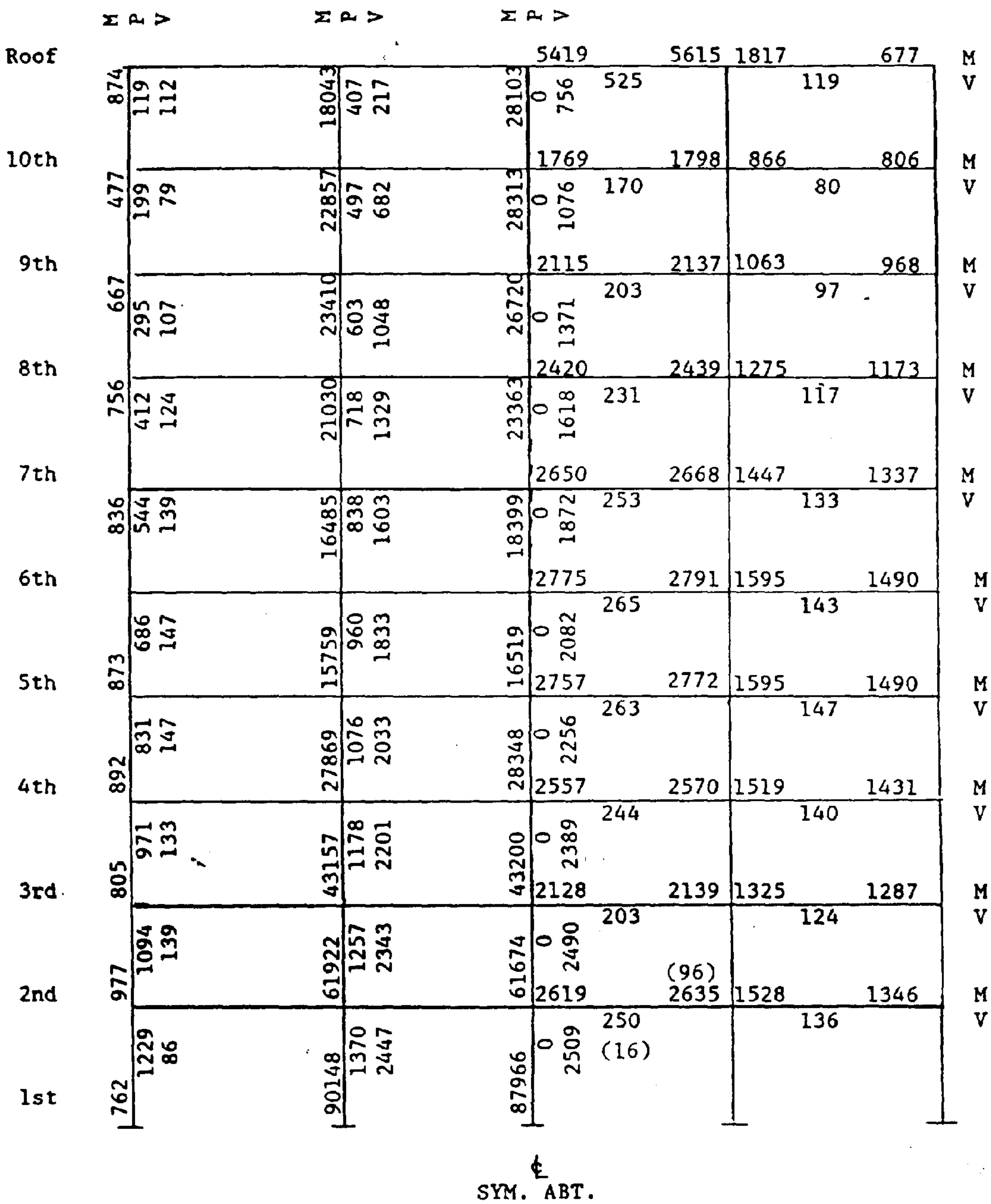
Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 27 of 32)

EQ II ELEMENT FORCES: LONGITUDINAL DIRECTION - FRAME LINES A & D

SEISMIC RESULTS FROM COMPUTER ANALYSIS.
UNITS ARE KIPS AND KIP-FT.



(DL + ½LL)

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 28 of 32)

"ACCIDENTAL" TORSION FORCES

The "accidental" torsion is the story shear, V_x, times the nominal eccentricity of 5% of the building dimension.

$$M_t = V_x \times 0.05 \times 190' = 9.5\ V_x$$

The story relative rigidity (K) of each shear element is obtained from the computer analysis.

$$\text{Torsional Shear} = \frac{Kd}{\Sigma Kd^2} \times 9.5\ V_x$$

$$\text{Direct Shear} = \frac{K}{\Sigma K} \times V_x$$

Distribution of Forces

9th Floor Level

SHEAR ELEMENT	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
1	19.99	94.5	1899	178516	$0.347V_T$	$0.043V_T$
2	0.35	67.5	24	1595	$0.006V_T$	$0.001V_T$
3	0.35	40.5	14	574	$0.006V_T$	$0.000V_T$
4	8.15	13.5	110	1485	$0.141V_T$	$0.002V_T$
5	8.15	13.5	110	1485	$0.141V_T$	$0.002V_T$
6	0.35	40.5	14	574	$0.006V_T$	$0.000V_T$
7	0.35	67.5	24	1595	$0.006V_T$	$0.001V_T$
8	19.99	94.5	1889	178516	$0.347V_T$	$0.043V_T$
Σ =	57.68					
A	16.77	40.25	675	27168	$0.472V_L$	$0.015V_L$
B	1.00	15.25	15	233	$0.028V_L$	$0.000V_L$
C	1.00	15.25	15	233	$0.028V_L$	$0.000V_L$
D	16.77	40.25	675	27168	$0.472V_L$	$0.015V_L$
Σ =	35.54		Σ =	419142		

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 29 of 32)

"ACCIDENTAL" TORSION FORCES

2nd Floor Level

SHEAR ELEMENT	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
1	29.66	94.5	2803	264871	$0.285V_T$	$0.043V_T$
2	0.27	67.5	18	1230	$0.003V_T$	$0.000V_T$
3	0.27	40.5	11	443	$0.003V_T$	$0.000V_T$
4	21.90	13.5	296	3991	$0.210V_T$	$0.005V_T$
5	21.90	13.5	296	3991	$0.210V_T$	$0.005V_T$
6	0.27	40.5	11	443	$0.003V_T$	$0.000V_T$
7	0.27	67.5	18	1230	$0.003V_T$	$0.000V_T$
8	29.66	94.5	2803	264871	$0.285V_T$	$0.043V_T$
	$\Sigma =$ 104.20					
A	24.84	40.25	1000	40242	$0.481V_L$	$0.015V_L$
B	1.00	15.25	15	233	$0.009V_L$	$0.000V_L$
C	1.00	15.25	15	233	$0.009V_L$	$0.000V_L$
D	24.84	40.25	1000	40242	$0.481V_L$	$0.015V_L$
	$\Sigma =$ 51.68		$\Sigma =$	622020		

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Figure F-4. Building with concrete moment-resisting frames and shear walls. (Sheet 30 of 32)

ELEMENT STRESS CHECK

Wall Lines 4 & 5

Neglect accidental torsional forces; less than 5% of the translational forces.

FLOOR LEVEL	V_D kips	V_u kips	$\frac{V_D}{V_u}$	SHEAR IDR	M_D ft-kips	M_u ft-kips	$\frac{M_D}{M_u}$	MOMENT IDR
8^{th}	1130	1050	1.08	1.75	25729	15350	1.68	3.00
1^{st}	3269	2710	1.20	1.75	299234	107080	2.79	3.00

Frame Lines A & D

Neglect accidental torsional forces; less than 5% of the translational forces.

At 1^{st} Floor Level

MEMBER ELEMENT	V_D kips	V_u kips	$\frac{V_D}{V_u}$	SHEAR IDR	M_D ft-kips	M_u ft-kips	$\frac{M_D}{M_u}$	MOMENT IDR
Wall	2447	1420	1.72	1.75	90148	37740	2.39	3.00
Beam	266	281	0.95	1.75	2731	1712	1.60	1.75

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 31 of 32)

CONCLUSIONS

The structure, as modified by the upgrading concept, will conform to the acceptance criteria for EQ-II forces; however a verification of soil capacities will be required as stated on sheet 21. It should also be noted that the detailed analysis of the existing structure (without modifications) indicates that the building has good overall performance characteristics, will remain essentially elastic for EQ-I, and would satisfy acceptance criteria for an earthquake slightly smaller than EQ-II (refer to sheet 16). Because this building is not an essential or high risk facility, the need for upgrading would be set at a relatively low priority as a result of formulating a decision by means of a cost-benefit analysis.

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Figure F–4. Building with concrete moment-resisting frames and shear walls. (Sheet 32 of 32)